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Section II - Subject 5

BEACH PROTECTION AS PART OF THE HARBOUR EXTENSION AT ZEEBRUGGE - BELGIUM

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Photograph 1 : General view of replenished beach east of Zeebrugge

RESUME

En 1976, le gouvernement belge a décidé de construire un nouvel avant-port à Zeebrugge (figure 1). Le nouveau port extérieur s'étendra de 3.5 km en mer.

Le port sera construit dans une aire du littoral très sensible à érosion et d'une morphologie complexe caractérisée par des forts courants de marée (jusqu'à 1,5 m/sec), des fines matières de fond et des bancs de sable, de vase et d'argile. La perturbation hydraulique causée par des travaux d'une telle ampleur, aura des répercussions sur la morphologie des environs.

Déjà à l'heure actuelle, la partie de la côte se trouvant au nord-est du port existant pose des problèmes. A cet endroit se trouve, à 500 m de la digue, un chenal dénommé "Appelzak" (figure 2) dont le fond se trouve à la côte H - 8 m (H étant le niveau moyen des basses basses mers devives eaux). Au cours des années, ce chenal s'est approché de la côte ; il est actuellement arrêté par des épis qui avancent jusqu'à 400 m de la digue. Depuis la plage, un transport latéral se produit vers ce chenal, d'où le sable est emporté parallèlement à la côte, par les courants de flot (figure 3).

Dans la zone de déferlement, donc plus vers la terre, le courant le long du littoral provoqué par les vagues fait sentir son influence ; dans la situation de 1976 cette influence était cependant affaiblie partiellement par les épis.

Puisque la construction du nouveau port extérieur pourrait provoquer des modifications dans cette zone, les autorités belges ont décidé de réaliser une étude détaillée pour définir les effets de cette construction, sur cette partie du littoral.
La situation de la plage étant très critique - en 1976 il n'en restait qu'une bande étroite - on a décidé déjà en 1977 de réaliser un important remblai initial de sable de 8.5 millions de m^3, soit d'environ 1.000 m^3/m'.
De cette façon la plage était élargée d'environ 100 m ; cet élargissement était très important tant pour le tourisme que pour la défense de la digue de mer. Le sable nécessaire a été déblayé en mer au moyen de dragues suceuses à élinde traînante, et deversé dans un approffondissement du chenal d'accès à l'écluse de Zeebrugge. De là, il a été repris par une suceuse à désagrégateur et refoulé sur une distance maximale de 11 km.

A cet effet on a installé sur la plage des stations de reprise (figure 10). Le sable utilisé avait un diamètre D_{50} variant entre 225 μ et 325 μ ce qui donnait lieu à une pente de la plage de 1:40 au-dessus et de 1:20 au-dessous de l'eau.

L'étude avait pour objet de définir l'érosion de la plage rehaussée et l'effet des défenses côtières sur cette érosion. Trois aspects différents sont examinés dans l'étude, notamment :

- l'érosion résultant des courants de marée
- l'érosion provoquée par le courant le long du littoral, orienté vers le nord-est
- l'érosion causée par le vent.

Pour faire un calcul le plus exact possible, on s'est servi d'une part pour déterminer les courants dans la situation future, de modèles hydrauliques (aussi bien physiques que mathématiques) et d'autre part on s'est basé sur des mesures sur place pour déterminer le transport actuel des sédiments, la nature du fond, le plan et hauteur des vagues, etc. Ces mesures ont du reste permis "d'étalonner" les formules de transport employés.

Les calculs ont alors été exécutés pour plusiers dispositions de la défense côtière, notamment la situation actuelle, la situation avec le nouveau port extérieur seul, et la situation avec le nouveau port construit mais accompagné de un ou deux épis submersibles avec crête se trouvant au-dessous du niveau de marée basse (figure 4).

Le résultat des calculs indique que l'érosion causée par les courants de marée ne sera pas modifiée par la construction du nouveau port extérieur ; en outre, elle ne nécessite pas de mesures directes.
Suivant les calculs, la construction d'un seul épi n'améliorera pas sensiblement la situation. Les calculs ont donné pour l'érosion dans le Appelzak, causée par des courants de marée, un volume de 400.000 m^3 par an, ce qui signifie pour la plage une érosion annuelle de 300.000 m^3.

L'érosion causée par le courant le long du littoral, provoqué par les vagues, a été évaluée à environ 430.000 m^3 par an ; on n'a pas trouvé raisonable de prendre directement des mesures pour y remédier p.e. sous forme d'une défense "dure". Ces mesures ne sauraient du reste pas être limitées dans l'espace, parce que les érosions se déplaceraient suite à ces mesures.

Puisqu'on a créé une plage sèche beaucoup plus large, l'érosion par le vent a une influence accrue. Elle a été estimée à 80.000 m^3 par an.

L'érosion totale ainsi calculée se monte à environ 800.000 m^3 par an sur une longueur de 8 km. Vu la grande sensibilité de ces calculs aux moindres modifications du régime hydraulique, un vaste programme d'observation a été conçu afin de détecter immédiatement des développements inattendus et d'y faire face en prenant les mesures appropriées.

1. BACKGROUND AND PROBLEM DEFINITION

The harbour of Zeebrugge is situated on the north-east coast of Belgium, some 10 km from the Dutch border (figure 1). It is located in a morphologically complex coastal region known as the Flemish Banks, on the seaward side of the Westerscheldt estuary. The morphology of this region is primarily determined by its relatively high current velocities (up to about 1,50 m/s at spring-tide) and the very fine sand and silt. Material transport is thus considerable, and morphological instability rapidly ensues from changes in the hydraulic regime.

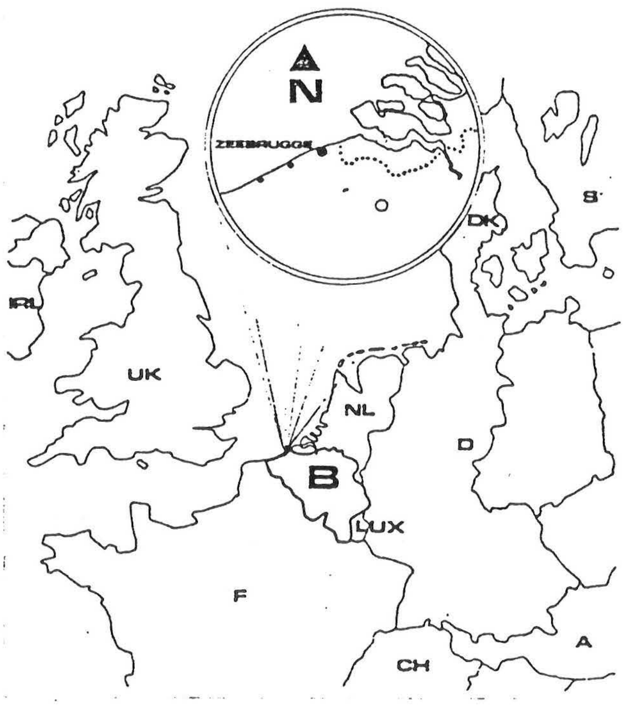


Figure 1

The present harbour of Zeebrugge is protected by a vertical breakwater more than 2 km in length which was constructed between 1897 and 1906.

East of the harbour and running in the direction of the Dutch coast and the Westerscheldt estuary is a trench called the "Appelzak" with a depth of H - 8 m (H corresponds to Low Low Water at Springtide). In 1900, when there was still hardly any harbour at all, this trench lay about 1 km seaward of the sea wall.

Construction of the harbour resulted in reduced sediment transport, particularly close inshore. This caused the Appelzak to shift ground and re-establish itself just off the coast, regardless of maintenance spoils that were regularly dumped into it. By 1976 a trench depth of H - 8 m occured just 500 m from the sea wall (figure 2).

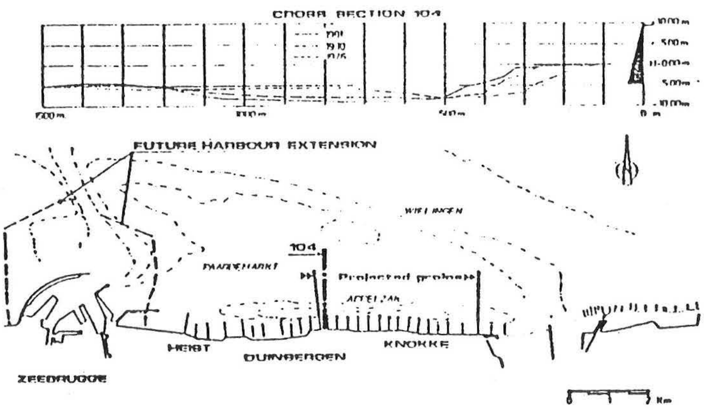


Figure 2

These problems affecting the east coast were discussed by Verschave at the International Navigation Congress in 1957 and the Coastal Engineering Congress in 1960.

As a form of protection for the coast, a large number of groynes had meanwhile been constructed each projecting about 400 m out from the sea wall. At present the Appelzak lies just seaward of these defences, prevented from still further migration inshore. Nevertheless, the following phenomenon occurs : beach material is transported offshore by wave action into the Appelzak from witch it is carried away by the predominantly northeasterly-flowing tidal current (see fig. 3). Consequently, by 1976, only a very narrow beach remained (see photograph 2).

Normally the interruption of the littoral drift engendered by waves is also a big problem in the extension of a harbour. However, since the littoral drift takes places in the breaker zone where the groynes inhibit sediment transport, this problem is lessened.

In 1976 the Belgian government commissioned the design and construction of the harbour extension at Zeebrugge to the T.V. ZeebouwZeezand (T.V. Z2), a consortium of eight contractors. The contract included to the planning and design of proper coastal protection for that section of the northeast coast.

The design phase established the optimal extension length of the new outer harbour, resulting in a total projected length from the coast into the sea of about 3.5 km. This would naturally bring changes in the hydraulic regime, which undoubtable repercussions upon the morphology of the coastal region concerned.

Provisional analysis of the problem had led to the conclusion that natural restoration of the beach was out of the question, so that this would have to be accomplished artificially by means of beach nourishment.

Already in 1977 - prior to the study - replenishment of about 8.5 million m^3 was decided upon for a stretch of 8 km, that is rather more than 1000 m^3 per running metre. This means widening the beach by about 100 metres, including in some places the construction of an entirely new beach.

The following sections of this report deal with :

- a detailed investigation into the consequences of extension for the coastal region concerned, and any measures that ought to be taken to protect the coast ;
- execution of the (already implemented) beach replenishment programme ;
- an observation programme aimed at ascertaining the accuracy of the study prognoses and enabling any new measures to be taken in good time.

2. STUDY

2.1. General approach

The study was aimed at investigating the probable consequences of harbour extension on the coastal region east of the harbour, followed by formulation of measures required for maintaining the replenished beach.

In view of the various types of erosion encountered here, the study can be divided into three parts (see figure 3) :

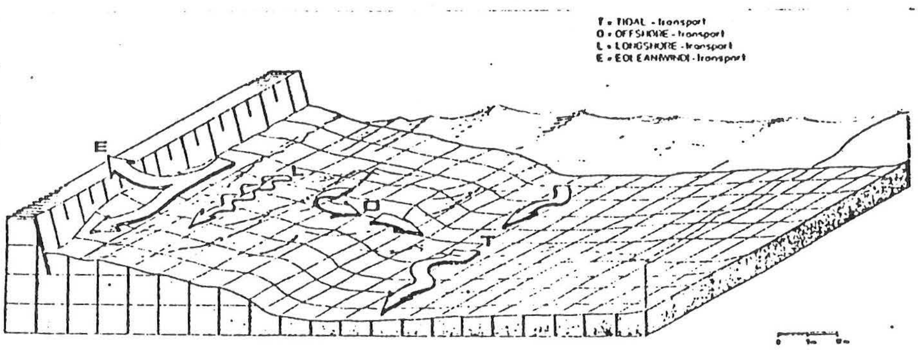


Figure 3 : different transport phenomena

- study of the morphological consequences of the changed tidal current pattern (T) resulting from the extension of the harbour, and the effect that this will have upon the beach (O)
- study of the wave-induced littoral drift, and the changes in this because of the harbour (L)
- study concerning erosion of the new beach by wind (E).

2.2. Sediment transport due to tidal currents

Physical and mathematical hydraulic models were used for this study. The hydraulic models were :

- a steady-state model with fixed bottom, horizontal scale 1:1000 and vertical scale 1:125 ; this model only included two periods for mean spring-tide, namely 40 minutes before H.W. (maximum flood stream) and 5 ½ hours after H.W. (maximum ebb stream)
- a physical tidal model with fixed bottom and the same scale as above
- a mathematical tidal model, used for calculating a mean spring-tide.

The physical models were constructed and operated by the Hydraulics Laboratory of Borgerhout, Antwerp, and the mathematical tidal model by the University of Liege.

A large number of in-situ measurements were also carried out, including :

- wave measurements in deep water at various points along the coast ;
- simultaneous current/turbidity measurements ;
- long-term sediment migration, using radioactive and fluorescent tracers ;
- bottom surface sampling, both on the beach, the underwater beach and in the Appelzak trench.

In the course of the study calculations were made of the morphological effect of various sorts of coastal defence upon the coast and the beach. The alternatives considered were :

- the existing situation without new harbour extensions ;
- the situation with new harbour extensions but without additional defence works ;
- the situation with new harbour extensions plus additional defence works, viz. :

* one downward sloping submerged groyne (below the low water line) about 1000 m long ;
* two downward sloping submerged groynes (below low water) about 1000 m long (see figure 4).

The object of calculating sediment transport due to tidal currents was to work out, for each alternative, what the consequences of the adopted measure would be for the coastal region concerned in terms of ad hoc sedimentation and erosion.

Ad hoc here, means the immediate values at the very moment when the physical alteration is carried out. The hydraulic regime will naturally be affected by changes in the bottom profile and this, in turn, will create a new bottom situation, until finally equilibrium is reached between current and material transport. For the time being these latter calculations are regarded as too speculative and they have not been carried out.

In order to calculate the erosion and sedimentation referred to, the coastal region in question was split up into a number of sections 500 x 1000 m^2 in area (see figure 4). The current was determined at the grid points of all these sections, which enabled sediment transport to be calculated, and after that quantification of sedimentation or erosion could be obtained from the balance between incoming and outgoing sediment transport.

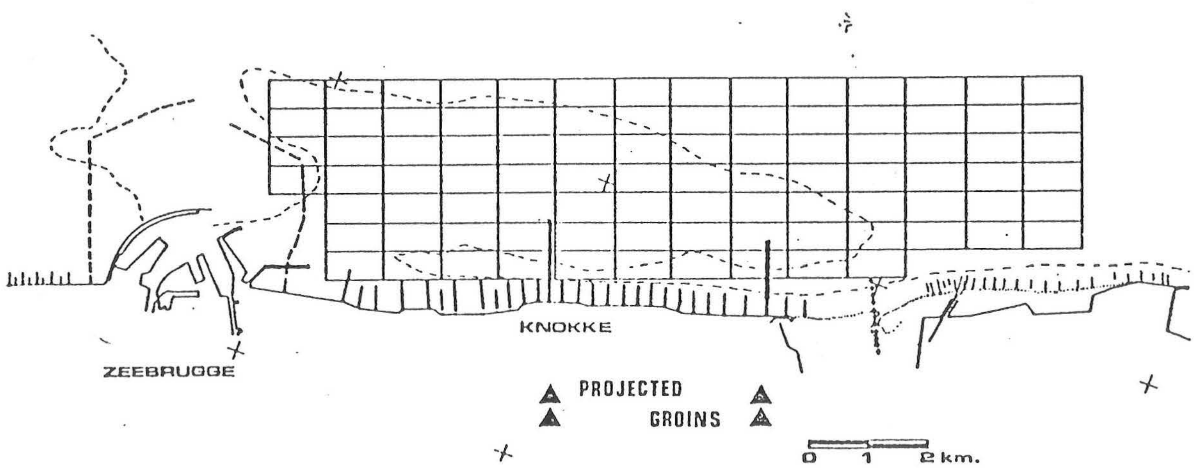


Figure 4 : sections used for transport calculations

The currents provided the most significant input data for the calculation of sediment transport. The latter was calculated during one tidal cycle at half-hourly intervals. From the hydraulic models, current velocities and directions were measured for each alternative system of coastal defence.

A difficulty with the steady-state model initially used for making the calculations was that only two tidal periods were known.
The other periods had to be calculated indirectly. An important item in this calculation is the application of a so-called water balance. Such a water balance also proves necessary in the case of a physical tidal model since there are always certain errors when measuring current velocities in such a model. The balancing method is used to level out the increases or decreases in the water level occurring mathematically (but not in reality) in certain sections.

Waves are an important additional factor in this coastal region ; consequently, to calculate sediment transport, use was made of Bijker's formula, which takes both currents and waves into account. The wave climate in the various sectors was determined with the aid of refraction and diffraction calculations, using as basic data the recordings taken over a period of many years on-board the lightship "Westhinder", stationed 30 km west of Zeebrugge.

It is common knowledge that the calculation of sediment transport due to to currents is always risk-prone - irrespective of the formula used. This applies even more in an area where sediment transport is largely determined by tidal currents, since then, the ensuing transport is actually the difference between two quantities and as we know, such so-called "differential" calculations are extremely sensitive to minor changes or inaccuracies.

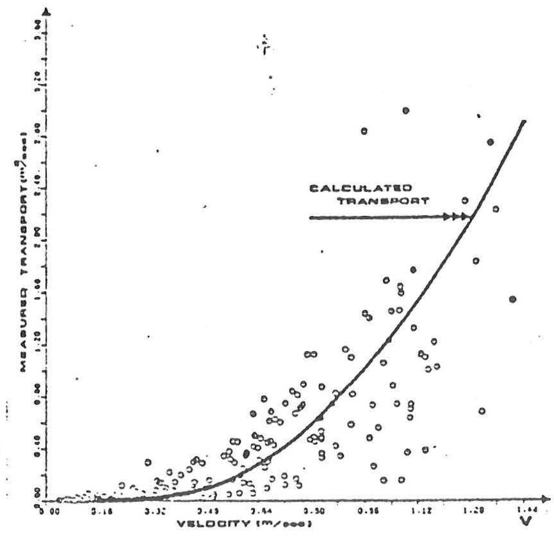


Figure 5 : comparison between measured and calculated transport

It was therefore decided to calibrate sediment transport due to currents by means of a combined current/turbidity measurement programme. In this way the various constants in Bijker's formula could be determined which meant that use could be made of a calibrated transport formula valid for this area. Figure 5 gives a comparison between measured transport and calculated transport. The correlation coefficient was 80 %.

Erosion and sedimentation were determined for the various alternatives in the manner described above.

All the calculations were carried out under a spring-tide and for five wave heights, followed by statistical processing to obtain an annual result (see figure 6). For the definitive annual result, a so-called tidal factor was determined which takes into account the effects that mean tide and neap tide have on the transport.

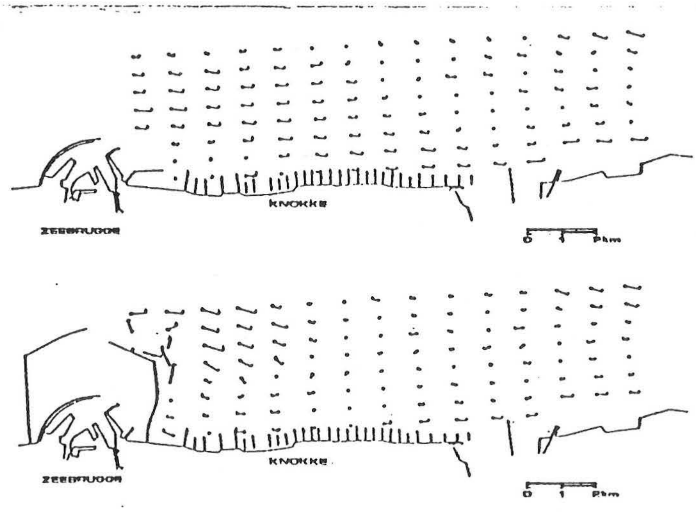


Figure 6 : examples of resulting sediment transport calculations

The results in terms of erosion as calculated for the Appelzak region are given in the following table :

	Steady-state model	Physical tidal model	Mathematical tidal model
Situation without new outer harbour	2.47	0.39	0.40
Situation with new outer harbour	2.85	0.50	0.40
New outer harbour plus one groyne	1.60	0.82	0.69
New outer harbour plus two groynes	1.19	not calculated	0.63

Table : Erosion, calculated for the various models in mil. m^3 per year (sedimentation not calculated)

This table clearly shows that there are large differences between the steady-state model, the physical tidal model, and the mathematical tidal model.
As stated earlier, these differences are due to the extreme sensitivity of these calculations to current data.

The physical tidal model was selected as being the most authentic, but obviously the results here must also be treated with considerable caution. The extreme sensitivity of the calculations lies in the fact that a velocity vector error of 5 cm/s, for example, can give a complete reversal of the erosion and sedimentation pattern.

This is understandable when it is realized that the resulting sediment transport, for instance, is the difference between a flood stream of 1.00 m/s and an ebb stream of 0.90 m/s. The error in resulting sediment transport will already be very large with the given velocity vector error of 5 cm/s, but if this error is made at two adjacent grid points, the error in erosion and sedimentation will be even larger. The values in one section are determined by the differences in resulting transport, in other words by differences within differences.

Once the current vector is correct, the possible error in actual sediment transport is much smaller, since it is known from correlation between measured and calculated transport. A correlation coefficient of 80 % is only a moderately good result, but since the total annual sediment transport is made up of a large number of separate variables, the calculated total annual transport may still be regarded as reasonably accurate due to mutual cancellations.
As the tests with the hydaulic models are forecasts of new situations, the errors are essentially unknown.

2.3. Translating tidal current erosion into beach erosion

The statistics presented in the foregoing paragraphs only relate to the erosion and sedimentation of the present Appelzak trench just offshore. Basically, however, the Coastal Authorities are not interested in this erosion but in that of visible coastline. The Appelzak erosion must therefore be represented in terms of beach erosion.

As stated earlier, the position is as follows : Tidal currents continuously transport material laterally out of the Appelzak, making it deeper. Migration of the trench closer to the shore is prevented by the existing groynes, making it steeper.
The instability of the submerged beach encourages transport offshore into the Appelzak, where it is taken away by the tidal currents. The latter should be changed however by the futur harbour extension.

The most pessimistic assumption regarding this beach erosion is when the beach erosion and Appelzak erosion are identical, so that it would not be the Appelzak eroding but just the beach. This assumption, however, is not entirely realistic, as shown below. Figure 7 illustrates the line of argument ultimately adopted.

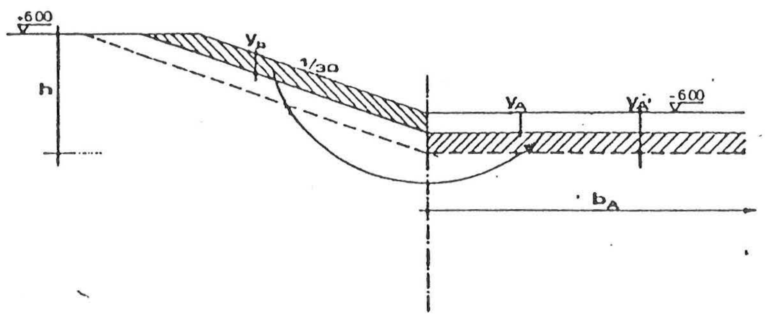


Figure 7 : schematic representation of Appelzak and beach erosion

It appears that sand with a D_{50} of about 250 u on the (meanwhile replenished) beach results in a stable beach slope of about 1:40 above water and 1:20 under water, i.e. an average of 1:30.

Should the Appelzak tend to erode by a quantity Y_A', it will be replenished from the beach but in such a way that a beach slope of average 1:30 will be sustained, while the ultimate lowering of the beach line Y_b will be equal to the ultimate lowering of the Appelzak Y_A.

The real relation between Appelzak erosion and beach erosion can then be calculated if the width of the Appelzak b_A, to which the above hypothesis applies, is known. On the assumption that this width varied between 120 m and 320 m, an optimistic and a pessimistic supposition were established, as shown in figure 8.

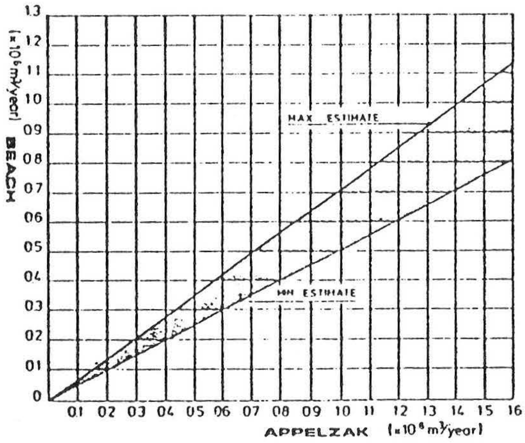


Figure 8 : relation between Appelzak and beach erosion

This enabled calculation of the quantity of beach erosion that may ultimately be anticipated. In the Appelzak both erosion and sedimentation occur, but only erosion was taken into consideration, on the grounds that sedimentation at one place does not eliminate the possibility of erosion somewhere else. Measures must always be taken to combat local erosion, even if 1 km further on, for instance, a great deal of sedimentation is taking place.

For the different alternatives under consideration, beach regression due to erosion by tidal currents is calculated to be as follows :

- present situation : 250.000 m^3/yr, or about 25 m^3/m'/yr
- new outer harbour : 300.000 m^3/yr, or about 30 m^3/m'/yr
- new outer harbour plus 1 groyne : 500.000 m^3/yr, or about 50 m^3/m'/yr

2.4. Calculation of wave-induced longshore drift and coastline morphology

Wave-induced longshore drift is primarily determined by waves from NW and WNW, which give rise to a drift current running north-eastwards, and waves from N which give rise to a coastal current running south-westwards.

The resulting sand transport thought to be due to this littoral drift ultimately proceeds north-eastwards and varies from place to place along the coast because of the varying refraction coefficients.
In the present situation, part of the sediment does get carried past the harbour breakwater because of the latter's configuration, but it does not return into the system of coastal transport eastwards of the harbour. Mean transport capacity eastwards of the harbour is calculated at 430.000 m^3/yr.

In the 1976 situation, prior to the implementation of the beach replenishment programme, the longshore drift was partly impeded by the existing groynes north-east of the harbour. The beach nourishment mostly covered the groynes with sand, so that they no longer form an obstacle to the littoral transport. Diffraction effects around the future harbour breakwater will cause transport capacity east of the future eastern harbour mole to build up gradually to about 430.000 m^3/yr. Supplying sand at this quantity will therefore be necessary in this area if permanent erosion is to be prevented.

A new obstacle to sand transport might be created by 1 km-long underwater groynes which is one of the alternative proposals for protecting the coast against erosion. The resultant blockage however, would only be very slight since the main purpose of these groynes is not to impede littoral transport but to rather reduce tidal currents and erosion in the Appelzak trench.

Coastline developments have been calculated with the aid of Pelnard-Considère's hypothesis that longshore transport is proportional to the angle of wave attack in the breaker zone. Two examples of coastline calculations are given in figure 9, based on the assumption that permanent artificial nourishment at the rate of 430.000 m^3/yr is taking place immediately to the east of the extended outer harbour. Figure 9a represents the situation with just the extended outer harbour, and figure 9b the situation with the addition of one groyne. It can be seen that some further erosion is occuring downstream of this groyne but that the coast by the Belgian-Dutch border is practically stable.

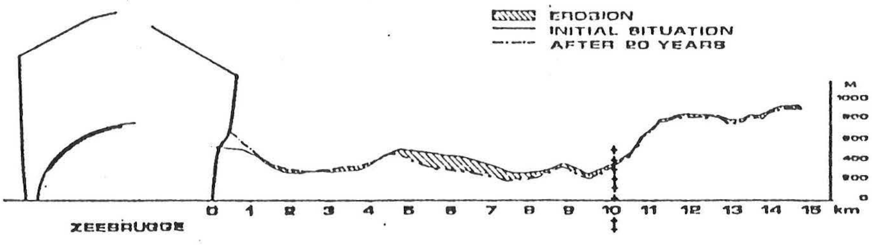


Fig. 9.a

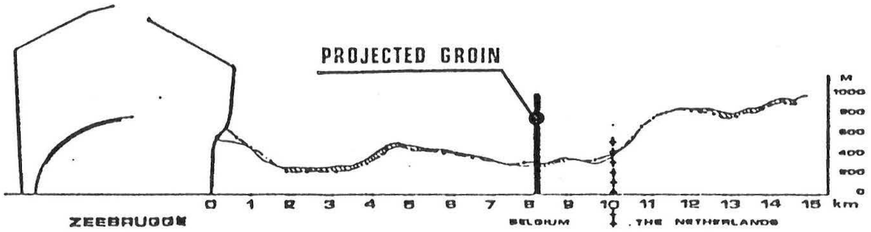


Fig. 9.b

Figure 9a, 9b : examples of coastline calculations

2.5. Wind erosion

On the new beach to the east of Zeebrugge, wind erosion will become more significant than it used to be because the replenishment will enlarge the beach by more than 100 m out to sea, so creating more dry sand.
Sediment transport by the wind was calculated using Bagnold's formula for sand in desert areas, i.e. with a moisture content of 0 %. Terwindt has done research into the relation between the moisture content on the beach and the minimum shear velocity needed to initiate sand transport by wind on the Dutch beaches. From this, sand transport by wind has been determined under various moisture conditions.

By combining these data with the climatological data, a wind erosion loss of 80.000 m^3/yr was finally calculated for the coastal region concerned. Most wind losses proceed inland (perpendicular to the coastline) and are a nuisance to the commercial establishments on the sea wall. Measures have therefore been sought to reduce these losses and further reference will be made to the subject in chapter 3.

2.6. Conclusions

In studying erosion problems a choice always has to be made between a hard defence - by means of groynes, for example, which are supposed to combat erosion - and a soft defence - implying that erosion losses are replenished at fixed intervals by materials equivalent to that lost. The selection of one system in preference to another is essentially a problem of economic optimalization, involving cost comparisons by capitalization of all (maintenance) costs.

It is known that hard defences merely displace problems to other locations ; for this reason a downward-sloping underwater groyne was considered as alternative protection for this stretch of coast. The decision should only really turn to a "hard" coastal defence if this has obvious advantages for the entire coastal area concerned.

The calculations made with the physical tidal model and the mathematical tidal model showed that, with an extended outer harbour, erosion due to tidal currents would remain approximately the same as at present but that the construction of one groyne close to the Belgian-Dutch border could cause some deterioration.

It followed from these calculations that a total annual beach erosion in the order of 800.000 m^3 must be anticipated.

For the time being the Belgian Authorities are adopting the view that the construction of the new harbour will not lead to any appreciable deterioration in the situation. In vieuw of the great importance of this coastal region and of its vulnerability, a detailed programme of observations has been drawn up (see section 4) so that measures can be taken in good time should intervention prove necessary.

3. EXECUTION OF THE BEACH NOURISHMENT PROGRAMME

3.1. Choice of borrow area and method of work

As stated earlier, the beach improvement works were carried out within the framework of extensions to the outer harbour of Zeebrugge. This meant that a number of data were obtainable from drillings made earlier in connection with the selection of a new navigation route to that harbour. This was a big help in finding suitable borrow locations for the more than 8 million m^3 sand required as beach fill.

For economic reasons, it was decided to deepen out a 5 km-long stretch of the navigation channel to the Westerscheldt, north of the "Wandelaar" sandbank, at about 20 km sailing distance from Zeebrugge. In the first couple of metres below the existing bottom, which varied there between H - 12 m and H - 15 m, sand layers were encountered with a mean grain diameter of D_{50} between 225 and 325 micron, a quality meeting all the requirements laid down for the beach improvement.

In view of the location of this borrow area, the use of large sea-going suction hopper trailing dredgers was an obvious choice. Not only because they enabled work to be continued under bad weather conditions for a comparatively long time, but also because they did not inconvenience shipping in any way.

These trailer dredgers were supposed to dump their loads in a special dumping pit. The approach to the new sea lock then under construction - which also has to serve as temporary work harbour for the new works in Zeebrugge -seemed suitable for this. From there, working continuously, a powerful cutter dredger and the necessary booster stations pumped the sand to the beach (figure 10).

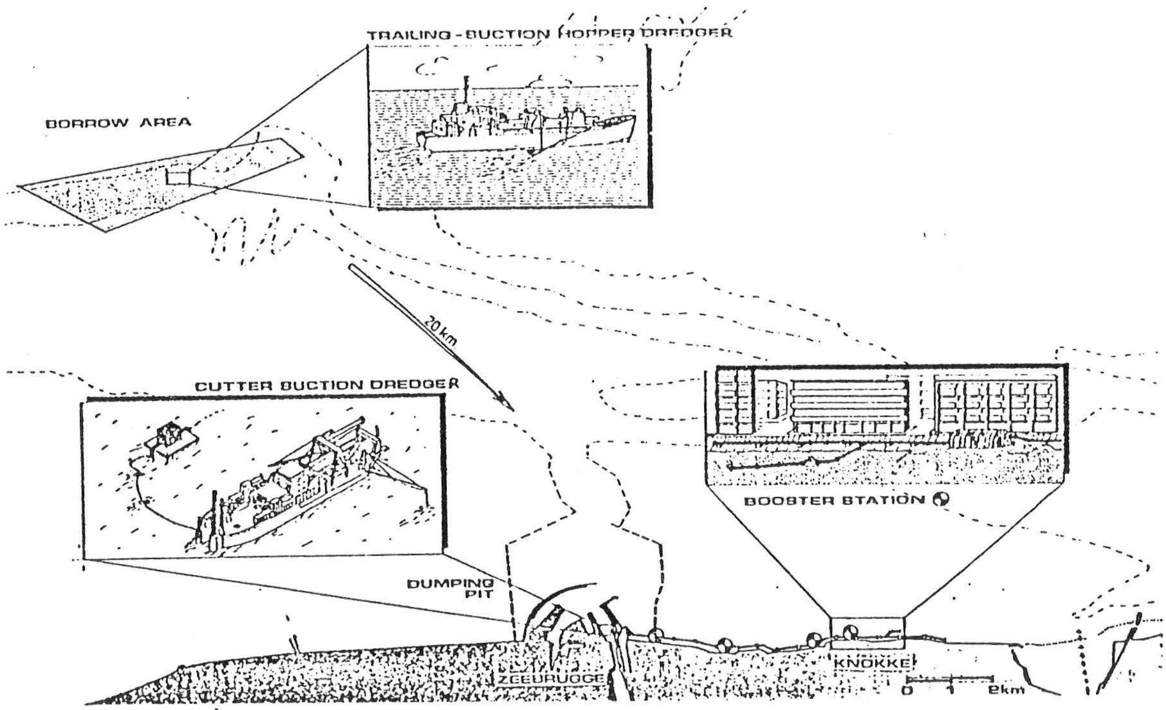


Figure 10 : general execution plan

To provide an initial protection and restauration for a 5 km-long section of the sea wall, it was decided to pump in about 2.600.000 m^3 of sand in the first phase. Subsequently in a second phase, about 5.800.000 m^3 sand was supplied, this time distributed over the entire length of the 8 km-long beach.

3.2. Execution, and plant employed

In December 1977 the approach to the new sea lock had been so far dredged that sea sand supply could be started.

The dumping pit was divided into two adjoining sections, each measuring 350 x 80 metres, so that the cutter could remain continuously at work. While one section was being dredged to about H - 15 m, the other was being filled to about H - 8 m.

The cutter dredger needed about 10 days, on average, to process one fully-dumped section.

The trailer dredgers had to be of appropriate size for the capacity of the cutter dredger, thus the hopper capacity of the vessels used for this operation varied between 5.000 and 7.000 m^3 sand. Fully loaded, they had a draft of 8 to 10 metres and could dump their loads at all times, irrespective of the state of tide. The average working cycle of a trailer dredger was about 4 hours per load : 1.5 hours dredging time, 2 hours sailing time, and 10 to 15 minutes dumping time.

At the end of 1977 the cutter dredger that had excavated the lock approach started the first phase of sea-sand supply. Initially, the sand was pumped over a distance of more than 6.000 metres via a 800 mm Ø pipeline.
To accomplish the operation three 2.500 hp. diesel booster stations were set to work astern of the cutter at 400, 1.300 and 3.400 metres respectively.
The first was a floating station near the end of the 500 metre-long floating pipeline, while the other two were situated on different parts of the beach above the high water line.

To bridge the greatest pumping distances (between 8.000 and 11.000 metres) two more booster stations were required, some 5.400 and 6.600 metres astern of the cutter dredger. Two 2.000 hp. electro-boosters were set up on the already partly replenished beach right opposite the centre of the seaside resort of Knokke-Heist. They operated on the municipal electricity network. Electric propulsion had been deliberately chosen to comply with the strict regulations governing noise abatement in this built-up and much frequented tourist zone.

In July 1978 the most distant point for beach reclamation was reached, involving a maximum pumping distance of 11.000 metres. This marked the end of the first phase, during which a volume of 2.600.000 m^3 of sand had been deposited over a length of 5 km, i.e. 520 m^3 per running metre.
Besides the cutter dredger with a maximum dredging depth of 28 metres, which was equipped with a 1.6000 hp. submersible pump and two 1.600 hp. pressure pumps, all five boosters were also in operation.
This meant that more than 16.000 hp. was being employed to pump an average mix concentration of 20 of 25 % in the 11.000 metre-long pipeline of Ø 800 mm at a velocity of 4.5 to 5 m/sec.

After that, working back again, the second phase of beach nourishment was effected, during which 5.800.000 m^3 of sand was distributed as evenly as possible over the 8 km of beach, i.e. an average of 725 m^3 per running metre. During this part of the operation the boosters were gradually put out of commission and in March 1979 the beach improvement was an accomplished fact.

It should be pointed out that the quantities specified above were determined by measuring the volume of each load in the holds of the trailer dredgers.
Retrospective measurements on the beach revealed about 80 % of these volumes in the beach profiles. About 15 % of the missing 20 % can be attributed to the greater density of the sand on the beach compared with that in the hold of the hopper, while the remaining 5 % may be due to finer grains swilling away as ground sea during the nourishment.

In this respect it is also interesting to compare the grain diagrams of a sand sample from the hopper and a sand sample on the replenished beach (figure 11).

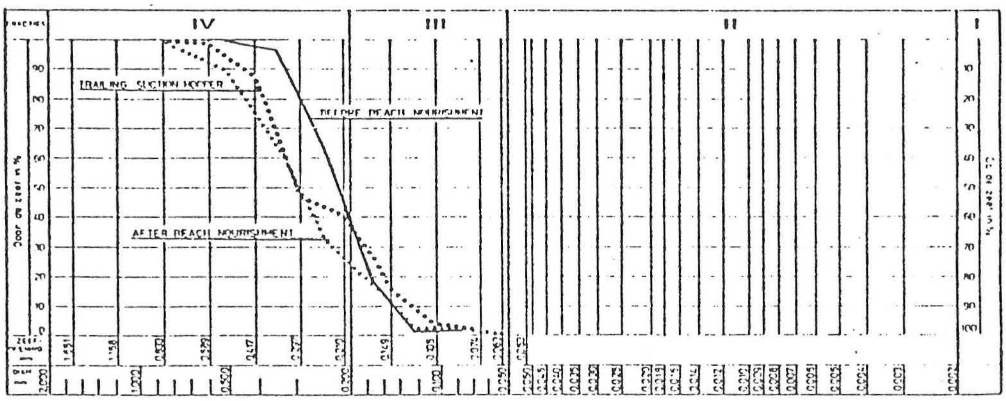


Figure 11 : grain size distributions

During the specified period the plant was in constant operation 120 hours a week, i.e. 24 hours a day, 5 days a week, from Monday to Saturday morning inclusive.
The necessary repair and maintenance jobs were done at the weekends.

3.3. Nourishment activities on the beach

The whole of the main pipeline was entrenched to a depth of one metre so that only the pipes actually spouting sand were in view over a short stretch of the beach. In fact, the operation turned out to be a tourist attraction since the good quality of the sand made it possible for people on the beach to make immediate use of the sand that had only been deposited the day before.

During the first phase two pipes supplied sand all along the sea wall to a width of 30 metres and a level of H + 6.0 m (see figure 12).

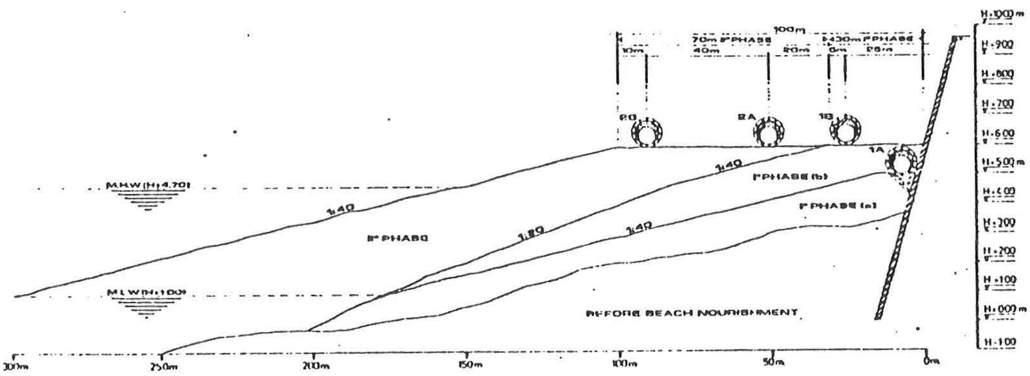


Figure 12 : first and second phase execution plan

Under low water conditions the first pipeline (1A) was used, which had been laid on concrete blocks specially placed at the H + 5.0 m level every 12 metres along the face of the sea wall. Under high water conditions the second pipe (1B), lying somewhat to the rear and 25 metres from the sea wall, simultaneously pumped sand over the first pipe, which later continued to function as the main pipeline.

This process was adopted because spouting underwater always creates a steeper face. Thus the pipe on the sea wall (1A), working under low water conditions, produces a comparatively gentle slope, while the second pipe, lying further out to sea, spouts a relatively steep slope under high water conditions. This means that the toe of the beach being replenished always forms at approximately the same place, sand losses are curtailed, and yet a sufficiently wide beach is still realized.

In addition, during the second phase, two other pipelines (2A and 2B) about 40 metres apart deposited sand along a following strip of beach, so that in the end a 100 metre wide beach was realized at a level of H + 6.0 m. At the request of the municipality of Knokke-Heist, a 25 metre wide strip was even nourished to H + 7.0 m for a distance of 2.000 metres along the sea wall. This was the busiest part of the beach and ensures that the properties there would be safe at high water.

As a rule, the newly-nourished faces, proved to take the slopes that where predicted : below the low water line an average of about 1:20 and above the low water line an average of about 1:40 (figure 13). This meant that at high water (H + 4.70 m) the beach had become 150 metres wide on average, and at low water (H + 1.00 m) it was actually 300 metres wide.

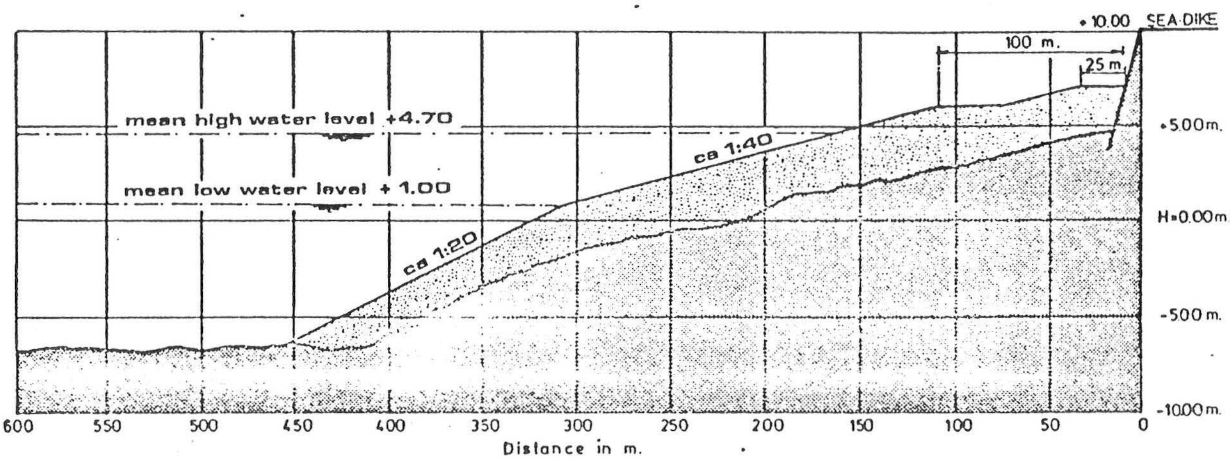


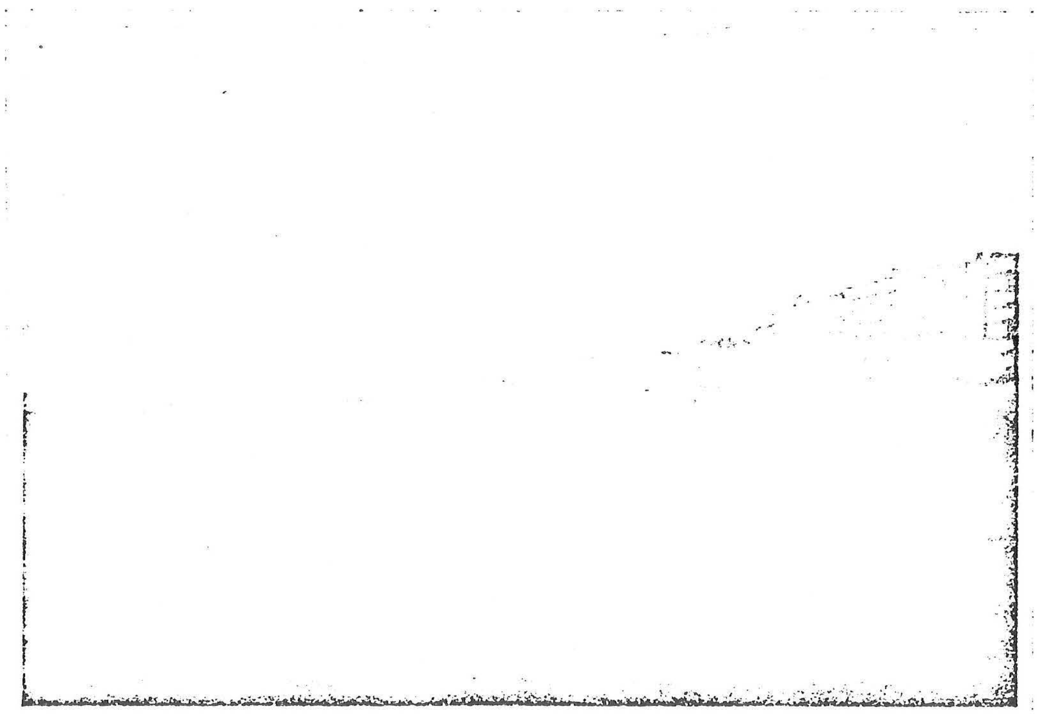
Figure 13 : Typical cross sections before and after beach nourishment

The complete implementation costs for this project amounted to about 2.000 million Belgian francs, which works out at 250.000 Belgian francs per running metre of beach, or about 1.500 Belgian francs per m^2 of beach above the high water line. The photographs 1, 2 and 3 show what was achieved by this investment.

2.000.000 m^3

=> 250 BEF/m^3

Photograph 2 : Knokke beach before restoration



Photofraph 3 : Knokke beach after restoration

3.4. Beach protection against wind erosion

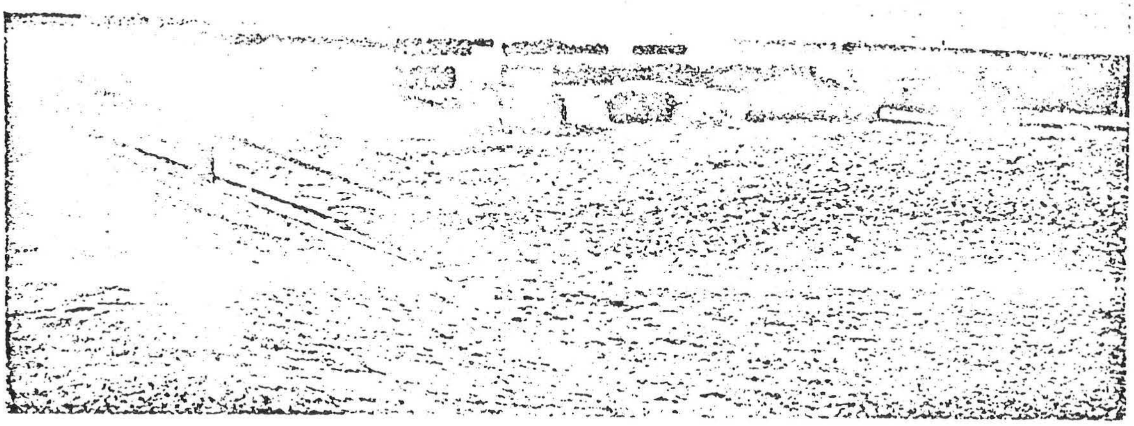
The definitive beach improvement resulted in a high and dry beach of about 150 m. This beach surface would be exposed to the predominant W and NW winds which would mean great quantities of whirling sand blowing along and over the sea wall.

Calculations were made of aeolian transport showing that sand losses could be in the order of 80.000 m^3 per year. In order to limit or prevent these losses, measures were sought for securing the stability of the sand immediately after replenishment.

Various methods were investigated, including the planting of vegetation, the implanting of straw, and the erection of windbreaks. The latter solution proved to be the most practical.

Two types of screens were ultimately employed - nailed wooden laths and osier hedges. The former are flexible and can be removed or repositioned once dunes have formed. The osier hedges can only be used once, but they are cheaper and require no upkeep.

The two types of wind screens were erected at locations where the sand can easily blow onto the sea wall or over it, particularly at those beach sections where there are no buildings. The screens have a porosity of about 50 % and are placed in rows perpendicular to and parallel with the sea wall. They are about 20 metres apart and openings are provided at regular intervals for the convenience of visitors. Initially these screens were placed on a few sections of the beach by way of experiment (see photograph 4). In practice it turns out that the hedges and screens have a most beneficial effect on beach stability. Small dunes form around the hedges. Moreover, they help to reduce sand nuisance on the sea wall.
Therefore, in future, similar screens and hedges will be placed on other sections of the beach as well.



Photograph 4 : Beach after restoration and with osier hedges

4. EAST COAST OBSERVATION PROGRAMME

4.1. Establishment of original situation prior to beach replenishment

Within the scope of the overall design for harbour extensions and beach nourishment, numerous measurements were carried out to record the situation before commencement of the works (To-situation). Beach, submerged beach morphology and sediment transport mechanisms off the coast, were measured in this connection by means of many observations and the application of various techniques. These included :

- Aerophotogrammetry : by means of aerial photographs instantaneous shots were made of the beach sections east and west of Zeebrugge. The advantage of instantaneous shots is that the entire coastal area is registered at one particular time.
  The measurements are as accurate as those obtained by levelling.
  The adjacent Dutch beaches were measured as well.
  The total survey area covers about 23 km of beach and dunes.

- Tracer experiments : the sediment transport mechanism off the coast and along the beaches was measured with the aid of tracers. Fluorescent tracers were injected at 9 locations in order to have an idea of longshore transport within the breaker zone before beach replenishment.

  Sand movements due to tidal currents were followed over long periods with the help of radioactive bottom tracers. Activated samples with the same granulometry as the local sand particles were inserted at various locations. Resultant transport can be established from these tests. These measurements were also coordinated with other sediment transport measurements and calculations.

- Turbidity measurements : in addition, to obtain a representative picture of current and transport movement prior to commencement of the works, an elaborate simultaneous survey programme was carried out with 11 survey vessels.

  At each location during one full tide (13 hours), at roughly 30-minute intervals, the current vertical and the sediment content vertical were measured. In all, more than 1.500 water samples were taken and 2.500 current measurements registered. At the same time detailed vertical tide recordings were proceeding at 5 locations along a 12 km strip of the coast. In addition, measurement buoys were placed in position to record wave characteristics.

  From the data, calculations were made of instantaneous and resultant transport per tide with a view to calibration of a sediment transport formula.

- Soundings : in correspondance with beach profiling, soundings were made along measuring lines established at right angles to the coast over a surface area of about 50 km^2. The measuring lines extended seawards to a distance of 2 to 4 km off the coast.

- Bottom sampling : to obtain satisfactory information about the bottom surface layer, more than 400 shipek bottom samples were taken along the study-area. The samples yielded the sand and silt content as well as the grain distribution of the sand particles. On the beach too, samples for comprehensive analysis were taken, along measuring lines at set distances from one another.
  The data obtained were used to compile charts showing the sedimentological characteristics of the surface layer.

- Wave measurements : use could be made of the recordings registered over a period of many years by the lightship "Westhinder" anchored in the deep-water zone off the Belgian coast. These data were supplemented with wave recordings from measuring buoys. Two buoys were anchored in the vicinity of Zeebrugge for automatic transmission of wave recordings to the shore at regular intervals.

- Current measurements : the movement of tidal currents at various depths was followed during one full tide with the aid of "floats" at several measuring points off the coast.

All these measurements gave a comprehensive picture of the hydraulic and sedimentological characteristics in the coastal zone concerned.

4.2. Further observation programme following beach replenishment

Nearly all the measurements listed in the foregoing paragraph are repeated regularly in order to provide a long-term understanding of beach and coast dynamics.

- The aerial photogrammetric observations take place twice a year and an additional observation of the coastal area in exceptional circumstances is foreseen.

- The soundings along set measurement lines are made twice a year and link up with the photogrammetric shots.

- Every month, 12 beach profile measurements are recorded to keep track of the beach evolution in the shortrun.

- Current measurements are carried out continuously at several measuring points to record any changes in the tidal current pattern in the Appelzak due to the harbour extensions.

- At several locations, simultaneous measurement campaigns are carried out annually over one full tide to establish whether changes are taking place in the transport mechanism. Moreover, in the future, additional measurements will be made during stormy weather in order to discern the effect of such extreme conditions.

- The wave recordings are carried out continuously at two locations off the coast.

- Bottom sampling takes place every 3 years to investigate long-term sedimentological changes.

- The beaches are sampled annually along various standardized measurement lines.

These measurements will not only permit verification of the calculations, but will also give the Belgian Authorities a chance to take action in good time for satisfactory coastal protection subsequent to the harbour extension.